Experimental Validation and Analysis of a CFRP-Retrofit of a Two-Column Bent

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ABSTRACT

Two identical ¼ scale models, representing typical bridges built in California before the 1970's, were built and tested using shake tables at the University of Nevada, Reno. An as-built bent was tested first and presented a brittle shear failure of the cap beam. A retrofit was designed implementing carbon fiber reinforced plastic (CFRP) fabrics to enhance the observed seismic behavior of the bent. The retrofit was designed to provide sufficient clamping force to enhance column lap splices, column and beam shear capacity, cap beam flexural capacity and beam-column joint shear. Experimental results proved the CFRP retrofit to be very effective, increasing ductilities from 2 for the as built to 7, and the performance of the bent was controlled by plastic hinging at the top of the columns. The effectiveness of the CFRP retrofit was evaluated in terms of the overall performance of the bent and of the enhancement of the deficiencies observed on the as-built bent. Analytical studies were performed on the as-built and retrofitted bent. A simple stress-strain model for FRP-confined concrete was developed. The model was implemented in the pushover analysis of the retrofitted bent. Experimental and analytical results will be discussed.

Keywords: Bridges, columns, retrofit, composites, earthquakes

INTRODUCTION

The structural design codes and construction practices are constantly updated to include the effect of earthquakes and material properties as more information is obtained. Old constructions, as is the case of many bridges in highways in California, must be retrofitted to achieve the current levels of lateral force capacities and displacement ductilities. The California Department of Transportation (Caltrans) has been developing and implementing a retrofit program for bridges that are seismically deficient. The use of fiber reinforced polymer (FRP) jackets is considered in this retrofit program.

Caltrans has developed preliminary design recommendations[3] for steel and FRP jackets, based on results of an extensive experimental program[4,6,14,15]. These studies proved the effectiveness of FRP fabrics for the enhancement in ductility, energy dissipation, lateral load carrying capacity, and ductile failure modes. However, the majority of the studies were limited to circular columns, and issues such as the retrofit of joints and lap splices have not yet been fully studied.

The presented research was conducted to study the seismic performance of typical two-column bents with drop cap beams built in California before the 1970's and to develop and validate experimentally the retrofit measures. CFRP fabrics were used to retrofit the columns, beam, and joint regions to achieve a ductile failure mode and adequate displacement ductility.

AS-BUILT BENT

Typical deficiencies of bridges built before the 1970s included insufficient longitudinal reinforcement of the columns, with lap splices at the bottom of the columns. The development length

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provided in the lap splices is approximately 60% of the required by current design guidelines[2]. The beam was also deficient, with longitudinal reinforcement detailed following gravity moment distributions and no joint reinforcement. The confinement reinforcement of columns and beams are excessively spaced and are open with 90^o hooks.

Typical pre-70 bridges built in California were studied. A prototype was developed with the average properties of the studied bridges. A ¼ scale was selected to fail the models subjected to seismic loading using shake tables. The models maintained the same properties as the prototype and construction details as those used in the 1970s. Figure 1 shows the overall dimensions of the bent and the dimensions and reinforcement of the columns and cap beam.

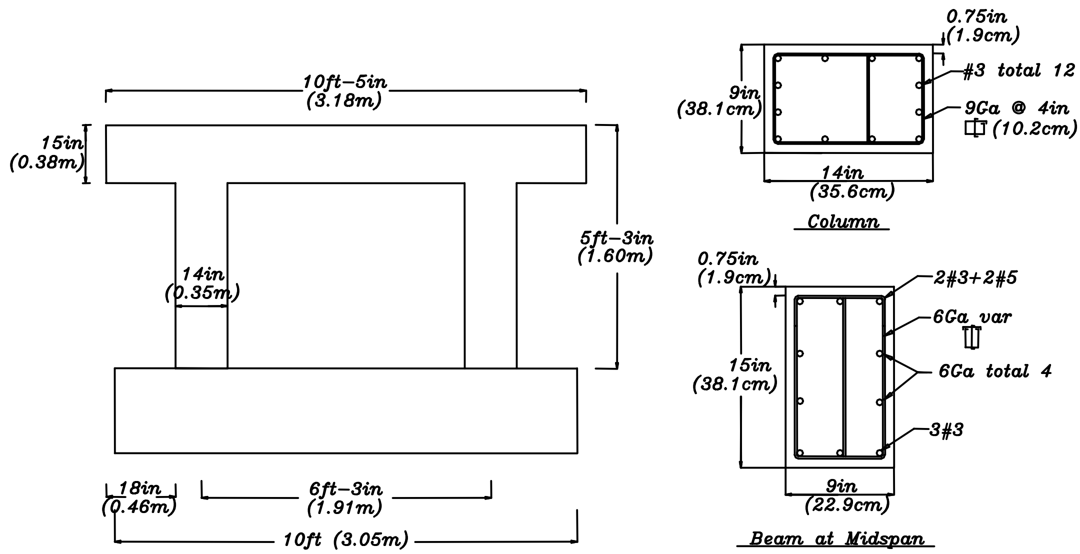

Figure 1 Test Specimen Dimensions

CARBON FIBER RETROFIT

The CFRP jackets were designed following Caltrans design guidelines[3]. Master Builders Technologies, a system that has been previously approved by Caltrans as an alternative column casing system, provided the CFRP fibers used. The design ultimate tensile strength in the fiber direction is 555 Ksi (3826 MPa), with a tensile modulus of the primary fibers of 29.2×10^3 Ksi (201.3×10^3 MPa) and a dry fiber thickness of 0.0065 in (0.165 mm) per layer, as specified by Caltrans. The retrofit was designed to provide additional confinement in the lap splice zone of the columns, flexural enhancement of the beam, and shear enhancement of the columns, beams, and joint. The column retrofit consisted of wraps with different number of layers throughout the height. The beam was retrofitted for shear with wraps and flexural enhancement was provided by longitudinal fibers on the sides of the beam. In a real bridge, the longitudinal girders would be supported on the bent cap, making it difficult to install rectangular CFRP wraps. At these locations, U-shaped wraps were used, and closed hoops that went around the entire section were used elsewhere. The beam-column joint region was retrofitted to enhance the shear capacity with vertical and horizontal fibers. The retrofit scheme is shown in Fig. 2.

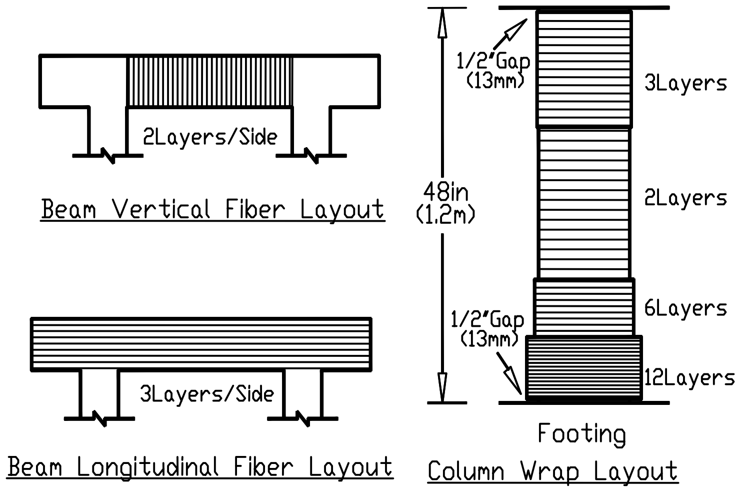

Figure 2 CFRP Layout for Retrofitted Bent

EXPERIMENTAL RESULTS

The bents were subjected to 1994 Northridge Earthquake, as recorded in the Sylmar Hospital. The time scale of the motion was modified by a factor of ½ to account for the quarter-scale size of the models. The peak ground acceleration of this motion is 0.6g. To detect the gradual deterioration and formation of plastic hinges, the specimens were subjected to increasing fractions of the Sylmar record, ranging from 25 percent to 225 percent of the motion. The loading protocol was identical for both specimens. Each bent was anchored to the deck of a shake table, as shown in Fig. 3. A steel transfer beam was designed to transmit the seismic forces into the bent, representing the same load transfer mechanism of the typical bents studied. The lateral loads were transmitted through an inertial mass system[6]. The vertical loads were applied through prestressed bars connected to hydraulic jacks.

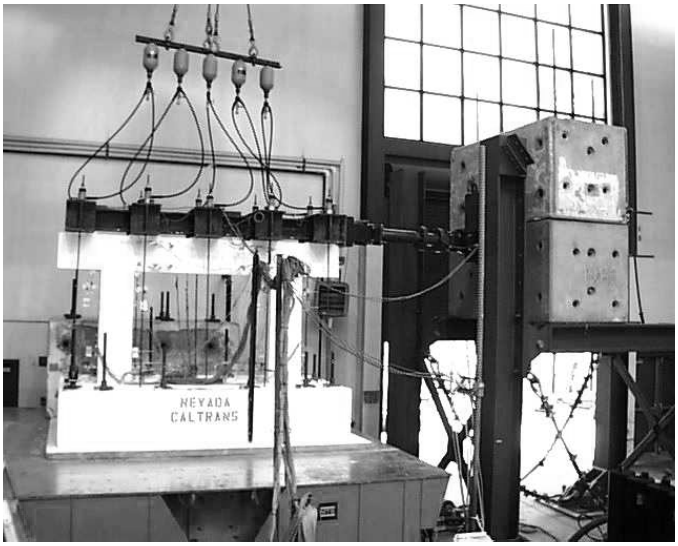

Figure 3 Test Setup

The as built bent (B2RA) had a brittle shear failure in the cap beam. Lap splice failure at the bottom of the columns began under low lateral loads. At a motion of 0.5xSylmar, corresponding to 0.3g, horizontal cracks were formed at the bottom of the columns, towards the end of the lap splice. These cracks were interpreted as the beginning of bond slip. Strain gauges placed in the longitudinal reinforcement indicated that the peak strains measured when slippage began were approximately 46%

of the yield strain. The main shear crack was first formed in the beam at 0.75xSylmar. Under this motion, the beam-column joint also cracked. Cracks extended to the tops of the columns at 1.0xSylmar, while horizontal cracks started forming at the bottom of the column, towards the end of the lap splice. The specimen continued to degrade until 1.75xSylmar, when loading was stopped due to considerable degradation of the beam and spalling of the concrete in the top and bottom of the column. Figure 5 shows the damage in the cap beam at the last motion.

Figure 5 Final Stage of As-Built Cap Beam

The CFRP retrofitted bent (B2RC) showed no sign of damage until 1.75xSylmar, during which horizontal cracks were seen in the composite at and near the top of both columns. The development length provided for the vertical CFRP fabrics in the beam-column joint had some effect in the right column, lowering the location of the crack. No effect was seen in the left column. The gap at the bottom and top of the columns were also cracked. No cracks were seen at the bent cap. The pier failed at 2.25xSylmar. Because the fibers were covering the structure, no gradual damage and crack formations could be seen in the concrete; the only evidence of damage was on the composite surface and on the gaps at the top and bottom of the columns. The failure was somewhat abrupt, although the measured lateral load indicated that the failure was imminent. There was a total separation between the bent and the footing by rupture of the column bars. Some of the starter bars were ruptured in a horizontal plane, while others showed evidence of necking. The left column failed at the top with rupture of some of the longitudinal reinforcement, while the right column failed towards the top of the column, at the end of the longitudinal CFRP fibers from the joint retrofit. The vertical fibers and transverse wraps on the right column acted as a composite section and debonded from the concrete. The cover concrete in the section of the crack spalled, although no evidence of damage in the core concrete was seen. The final stage of B2RC and a detail of the left column are seen in Figure 6. After the test was completed, the composite fibers were partially removed from the bent. The only cracks present in the concrete were in the zones where the composite had cracked. The beam remained intact, and the damage in the concrete was limited to the top portions of the column.

EFFECTIVENESS OF THE CFRP RETROFIT

The bent retrofitted with CFRP fabrics worked very well under the same earthquake loading as the as-built specimen. A comparison of the envelopes of the lateral loads vs. displacement for the bents is shown in Fig. 7. Current procedures used for developing backbone curves are not intended for specimens reaching peak forces at low displacements after the stiffness has started degrading, which is the case of specimen B2RC. The points that did not correlate well with the backbone curve are plotted to show the significant difference between them and the general trend of the force-displacement behavior of the CFRP retrofitted bent. Each curve was idealized by an elasto-plastic curve to determine

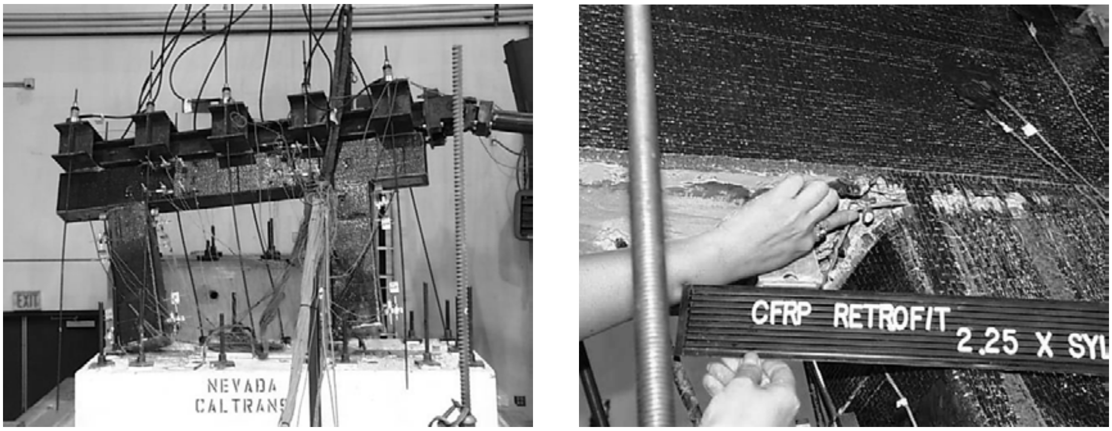

Figure 6 Final Stage of CFRP Retrofitted Bent

curve to determine the yield force and displacement and ultimate displacement ductility. The elastic part for each curve was obtained by connecting the origin to a point on the measured curve at which the force was one-half of the peak value. The yield force was established by equalizing the area between the measured and idealized curves. Failure was assumed to occur when the lateral load had dropped by 20 percent of the peak strength. The failure of the as-built specimen was considered to occur at 1.25xSylmar due to the presence of wide shear cracks in the cap beam. In reality, a bridge under these conditions would be closed and set for repair. There is a significant increase of the ductility, capacity, and stiffness in the retrofitted bents. The displacement ductility of specimen B2RA at failure was 2.2, while specimen B2RC achieved a displacement ductility of 7.4. The idealized lateral load at yield of the retrofitted specimens was 26% greater than that of the as-built bent.

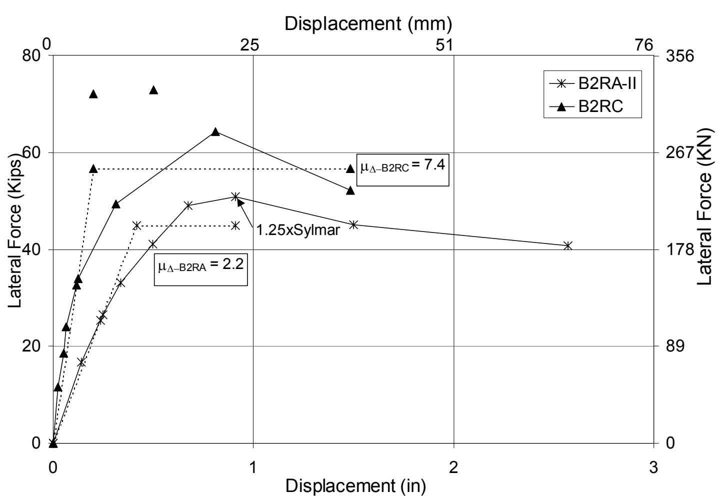

Figure 7 Comparisons of Force-Displacement Hysteresis Envelopes

PROPOSED SIMPLE FRP-CONFINEMENT MODEL

Several studies[5,7,9,16] have been reported on the stress-strain curve of FRP encased concrete. The FRP-confined concrete models generally consist of two linear segments that are connected by a curved transition zone. A minimum variation in the moment curvature properties was observed after performing the analysis of the column rectangular cross section using the existing confinement models. Furthermore, the differences among the models became even smaller when the moment curvature properties were implemented into the pushover analysis. Considering the negligible final difference between the FRP-confined concrete models used, the level of uncertainty of the behavior of CFRP fabrics when used to wrap reinforced concrete sections, and the complexity of the existing models, a simplified model was proposed.

The step-by-step procedure to determine the CFRP confined concrete stress-strain curve is as follows,

Step 1: Determine the stress at the break point, f_{co}, corresponding to an axial strain of 0.002, as follows

$$f_{co} = 57000\sqrt{f_c^{'}}\,\varepsilon_{co}\,(psi) \tag{1a}$$

$$f_{co} = 4700\sqrt{f_c^{'}}\,\varepsilon_{co}\,(MPa) \tag{1b}$$

where ε_{co} = strain at the break point, taken as 0.002, f'_c = unconfined concrete strength, and f_{co} = stress at the break point

Step 2: Determine the confinement pressure as described by the following,

For circular sections: $$f_r = \frac{2E_j\varepsilon_j t_j}{D} \tag{2}$$

For rectangular sections: $$f_r = K_e\left(\left(\frac{E_j\varepsilon_j t_j}{d}\right)+\left(\frac{E_j\varepsilon_j t_j}{b}\right)\right) \tag{3}$$

where E_j = elasticity modulus of the jacket, ε_j = strain of the jacket in the direction of the fibers, t_j = thickness of the jacket, D = cross section diameter, K_e = shape factor, taken as 0.75 for rectangular cross sections, and d, b = depth and width of the cross section

Step 3: Determine the peak confined stress as,

$$f_{cu} = f_c^{'} + 5.5 f_r^{0.7}\,(ksi) \tag{4a}$$

$$f_{cu} = f_c^{'} + 9.8 f_r^{0.7}\,(MPa) \tag{4b}$$

where, f'_c = unconfined concrete strength, and f_r = confinement pressure from the jacket

Step 4: Determine the ultimate strain by the expression,

$$\varepsilon_{cu} = \frac{\varepsilon_{CF}}{0.09 - 0.23\ln\left(\frac{f_r}{f_c^{'}}\right)} \tag{5}$$

where ε_{CF} = ultimate CFRP jacket strain, taken as 50% of the measured ultimate tensile strain of the CFRP, f'_c = unconfined concrete strength, and f_r = confinement pressure of the jacket.

A comparison of the stress-strain curve for the bottom portion of the retrofitted column with 12 layers of CFRP wraps was done using the proposed and reviewed models. The curves follow the same trend and have no significant difference in the moment-curvature properties.

PUSHOVER ANALYSIS

Detailed analyses were performed to correctly establish the force-displacement curve using analytical tools. Research and design oriented software[10,13] was used. The column moment-curvature properties were established implementing the simple model presented. Failure of the section was always caused by crushing of the concrete. The full yield moment was not expected at the bottom of the columns due to the insufficient development length provided at the lap splices. A yield ratio was determined as the actual development length over the required development length. This ratio was used to determine the maximum moment that could be reached at the bottom of the columns.

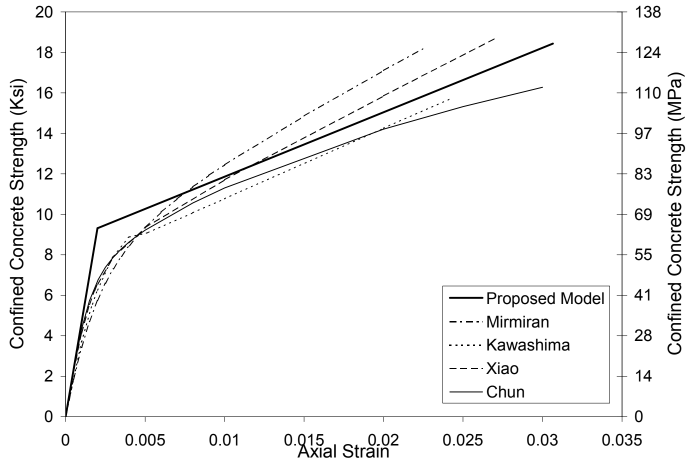

Figure 8 Comparison of Proposed FRP-Confined Concrete with Reviewed Models

The confinement of the CFRP wraps on the cap beam was neglected due to the open shape of the wraps. The longitudinal CFRP fibers were modeled as lumped fibers on the sides of the beam. The effect of considering the fibers located from the tensile most section of the beam to the neutral axis only versus the fibers along the entire height of the beam was studied. Due to the rigid properties of the FRP matrix once the fabrics have been saturated with epoxy, the M-ϕ relationships including the FRP fibers on the entire depth of the cap beam were used.

The almost elasto-plastic shape of the M-ϕ of the as-built bent cap changed significantly once the CFRP longitudinal fibers were included in the analysis. Once the longitudinal steel yielded, the CFRP controlled the M-ϕ curve, giving a high slope to the post-yield portion of the curve. Before the concrete reached the crushing strain, some of the longitudinal fibers ruptured, causing significant drops in the curve. Typically, a specimen is considered to have failed when the lateral load carrying capacity drops 20% relative to the maximum lateral force. In a consistent way, failure of the section was considered when the concrete reached the crushing strain or when the moment dropped by 20% relative to the maximum moment,

Figure 9 shows the comparison of the measured force-displacement curve for the CFRP retrofitted bent and the analytical results. The displacements obtained from the analytical programs included only flexural deformations. Shear deformations based on uncracked shear stiffness of the columns and bond slip deformations were included in the force-displacement curve. The ultimate displacement was calculated based on the summation of the displacement when four plastic hinges were formed, and the additional displacement that is necessary to fail any of the plastic hinges. The incremental displacements due to shear and bond slip are zero due to the elasto-plastic idealization of the M-ϕ curves for each structural element. The plastic hinge length was calculated using the only available formula for retrofitted columns[15], as follows,

$$l_p = g + 0.3 d_b f_y (Ksi) \tag{6a}$$

$$l_p = g + 0.044 d_b f_y (MPa) \tag{6b}$$

where g = gap between jacket and supporting member, d_b = diameter of reinforcement, and f_y = yield strength of the longitudinal reinforcement.

The calculated plastic hinge length resulted in a value of 8.1 in (206 mm) was obtained. A typical value for the plastic hinge length of an element without retrofitting is ½ of the section depth, which results in 7 in (178 mm). The calculated plastic hinge length seems very high considering the additional confinement pressure provided by the CFRP and the restraining effect it would have on the spreading of the yielding. Based on experimental results, the plastic hinge length was approximately 1.5 in (38 mm). This plastic hinge length was used to calculate the ultimate displacement of the retrofitted bent.

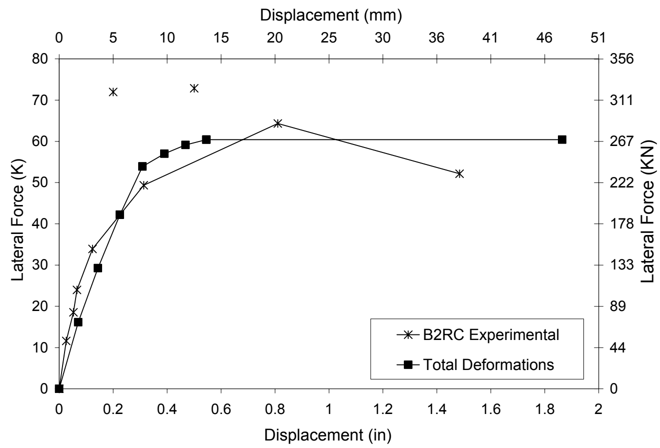

Figure 9 Force-Displacement Curve of the CFRP Retrofitted Bent

OBSERVATIONS AND CONCLUSIONS

Typical reinforcement and construction details used in bridges built previous to the 1970's are inadequate for seismic forces and must be retrofitted to meet current seismic design standards. The use of CFRP fabrics as a retrofit technique was successful in changing the failure mode of the as-built bent and concentrating damage to the top of the columns, reaching a lateral displacement ductility of more than 7 for the frame.

The use of U-shaped CFRP fabrics is effective in enhancing the shear capacity of a section and allows for easy installation without interfering with the superstructure of the bridge. Beam flexural capacity can be enhanced by means of longitudinal CFRP fabrics with sufficient bond length. The capacity can be determined by treating the CFRP as lumped fibers. The design of CFRP jackets must be done in a conservative way. Fifty percent of the ultimate strain of a CFRP tensile coupon should be taken as the maximum strain of the CFRP jacket. The strain limitations and required confinements by Caltrans should be implemented in the design, as they proved to be adequate for rectangular sections.

ACKNOWLEDGEMENTS

The research reported in this paper is sponsored by the California Department of Transportation and is being conducted at the University of Nevada, Reno under the advisement of professors Mehdi Saiidi, David Sanders, and Ahmad Itani. Assistance from the technical staff of the UNR bridge laboratory is gratefully acknowledged.

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